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1969

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W. A. WAHLER
& ASSOCIATES

PALO ALTO NEWPORT BEACH

W. A. WAHLER
& ASSOCIATES 1023 CORPORATION WAY, PALO ALTO, CALIFORNIA 94303 • (415) 968-6250 • TELEX 34-8321

April 8, 1969
Project 0276

Lloyd C. Fowler
Santa Clara County Flood Control and
Water District
516 E. Martha Avenue
San Jose, California

Attention: Mr. John L. Richardson

RE: STABILITY INVESTIGATION OF EMBANKMENT
COYOTE CREEK AT EMPIRE GARDENS SCHOOL
SAN JOSE, CALIFORNIA

Dear Mr. Fowler:

In accordance with your request of March 13, 1969 we have conducted a stability investigation of a 150 foot long stretch of Coyote Creek immediately adjacent to the Empire Gardens School. With no equipment surcharge applied, our findings indicate that the embankment is safe, although not excessively so, under its present 2-3/4 to 1 slope. Moderately heavy equipment (to 35 tons) would probably detect some weak spots along the top of the embankment, but be safe from gross failure; heavy equipment (70 tons or more) would fail.

The proposal to increase the slope to 2 to 1, and move the top of slope approximately 6 feet closer to the school would result in incipient failure or failure under all conditions studied by us. Hence, we recommend against increasing the slope inclination to 2 to 1 or translating the top of slope closer to the school.

Our subsurface exploration indicates that approximately 20 feet of loose sand fill with some intermixed debris has been dumped on the site to make grade for the school and adjacent yards. Loose granular fill will be subject to submergence settlement as the creek water level rises during a high flood stage. Earthquakes can cause seismic compaction of the loose fill which in turn can cause settlement damage to the school structures.

Lining the slope would not measurably increase the slope stability but would serve to reduce erosion of the loose granular slope soils. Laying the slope back to a 4 to 1 inclination or more, would increase the stability but require the relocation of the adjacent school structure.

The soil engineer assigned to this project was Mr. Anil Butail of our staff. If you have any questions on our subsurface exploration, conclusions, and/or recommendations, please call us.

Very truly yours,

W. A. WAHLER and ASSOCIATES

J V Lowney

John V. Lowney RE15774
Associate

JVL:eb
Copies:
Addressee (5)

TA710.3C69S72 1969

STABILITY INVESTIGATION OF EMBANKMENT

For
COYOTE CREEK AT EMPIRE GARDENS SCHOOL
SAN JOSE, CALIFORNIA

To
SANTA CLARA COUNTY FLOOD CONTROL and
WATER DISTRICT
516 E. Martha Avenue
San Jose, California

April 1969

W.A. WAHLER
& ASSOCIATES

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STABILITY INVESTIGATION OF EMBANKMENT
FOR
COYOTE CREEK AT EMPIRE GARDENS SCHOOL
SAN JOSE, CALIFORNIA

A. INTRODUCTION

This report presents the results of our stability investigation for the side slope of Coyote Creek next to the Empire Gardens School, San Jose, California. The purpose of these analyses was to evaluate the stability of the existing 2-3/4:1 slope and proposed 2:1 cut slope 6 feet closer to the school, and make recommendations concerning the slope stabilization, if necessary.

B. SCOPE

The scope of work performed in this investigation included a site reconnaissance, subsurface exploration, laboratory testing, engineering analyses of laboratory and field data, application of the data to the stability analyses, and the preparation of this report. The stability analyses performed were for the purpose of providing evaluations and/or recommendations regarding:

1. Stability of existing and proposed slopes.
2. Damage potential to the school.
3. Embankment and school protection.

C. SITE CONDITIONS

1. Surface The slope being investigated is a 150 foot stretch of the west bank of Coyote Creek at the Empire Gardens Elementary School in San Jose, centered at Station 615+00, approximately. At the time of our field exploration most of the slope was covered with vegetation. One of the buildings of the Empire Gardens Elementary School is located about 20 feet west of the top of slope. A chain link fence around the school runs along the slope leaving a path 5 to 10 feet wide on the top of the slope. The existing slope is irregular in section and at Station 615+00 has an approximate slope of 2-3/4:1.

2. Subsurface Information from the San Jose Unified School District revealed that the entire site of the school area and adjacent playground had once been the city dump. The soil to a depth of about 20 feet is a sanitary landfill consisting of loose sand and gravel, broken rock, and miscellaneous debris. This

material is highly heterogeneous in character but appears to be primarily a sand. Because it is loose it has a relatively low frictional strength for a granular material. The material underlying the fill is a gray, stiff, silty clay of moderate plasticity with reasonably good strength properties as determined by us by sensitive laboratory tests on a nearby project. Water was encountered at 28 feet in the boring.

D. STABILITY ANALYSIS

The embankment near the southeast corner of the school building is presently at a 2-3/4 to 1 slope, and under one proposal is to be cut to a 2 to 1 slope and moved approximately 6 feet closer to the school building. Based upon the results of the three borings drilled at the site and the standard penetration resistance encountered on the sampling spoon, we estimate that the friction angle on the loose granular fill (gravelly sand with some debris) ranges from 20 degrees to 25 degrees. Because of the heterogeneous nature of the fill it would not be meaningful to perform tests to determine the strength properties.

1. Method Three cases of applied forces were investigated by us; 1) steady state seepage, with the water level in the creek at 7 foot depth (typical winter condition); 2) a 35 ton shovel on top of the embankment, and 3) a 70 ton shovel on top of the embankment.

The rapid drawdown condition was considered but is believed inappropriate with pervious or semi-pervious materials such as the subject land fill, since the rate of pore pressure dissipation when the water level in the creek drops rapidly determines whether the rapid drawdown condition is pertinent or not or even to what degree it effects the existing stability. For instance in a relatively impervious clay the rapid drawdown condition is completely in effect, since the clay permeability is so high that drainage of the pore water can not take place for extended periods of time. The effective driving force on the slope is increased by 62.5 pounds per cubic foot. However, for the sand fill under study here, drainage of any water resulting from high flood levels would be rather quick, and hence, the rapid drawdown condition would not be nearly so influential in reducing the static factor of safety.

With granular materials the resistance to driving forces is approximately reduced by 50 percent by virture of being submerged in a flood stage. Hence, instead of weighing approximately 110 to 120 pounds per cubic foot, the effective soil weight is approximately 50 to 60 pounds per cubic foot during flood submergence, which in turn would reduce the factor of safety considerably below its equivalent static condition. Due to the high permeability of the material this condition would be reflected by a steady state seepage condition with high flood water levels in the fill; this condition was checked by us (factor of safety = 1.03 at 20°).

2. Results The factor of safety was determined by three methods: 1) conventional method of slices; 2) the Bishop Method; and 3) the Janbu Method. Using two cases of the friction angle, 20 degrees and 25 degrees, which we expect to be the limiting range, the factors of safety were computed for the above three cases by the three methods. Appendix C, Stability Analyses, contains a detailed description of the various methods used. . The existing and proposed slopes were examined with the results as shown on Table 1, below, Summary of Stability Analyses.

TABLE I - SUMMARY OF STABILITY ANALYSES

I Estimated friction angle of sand/debris fill = 20°

		FACTOR OF SAFETY		
SLOPE	CASE	NO. 1	NO. 2	NO. 3
Existing (2-3/4:1)	Steady state w/@ Elevation 63'	1.04	1.17	1.15
	Steady state water @ Elevation 76'	1.03		

TABLE I (cont.)

SLOPE	CASE	FACTOR OF SAFETY NO. 1	NO. 2	NO. 3
	35 ton shovel on top of slope	0.89	--	--
	70 ton shovel on top of slope	0.72	--	--
Proposed Cut (2:1)	Steady state	0.77	0.81	0.81
	35 ton shovel on top of slope	0.59	--	--
	70 ton shovel on top of slope	0.54	--	--
II Estimated friction angle of sand/debris fill = 25°				
Existing (2-3/4:1)	Steady state water @ Elevation 63'	1.33	1.50	1.47
	35 ton shovel on top of slope	1.14	--	--
	70 ton shovel on top of slope	0.92	--	--
Proposed Cut (2:1)	Steady state water @ Elevation 76'	0.99	1.04	1.04
	35 ton shovel on top of slope	0.75	--	--
	70 ton shovel on top of slope	0.69	--	--

Factor of Safety No. 1 - Conventional Method of Slices.

Factor of Safety No. 2 - Bishop Method - side forces between slices neglected.

Factor of Safety No. 3 - Janbu Method - side forces between slices considered.

3. Evaluation As can be seen from the above, the existing slope under the normal winter condition (steady state seepage) of about 7 foot creek depth, and as determined by the Bishop Method, resulted in a factor of safety of 1.15 which is not high. With the creek at the 100 year flood level the factor of safety is reduced to 1.03 using the conventional method of slices.

With the proposed 2 to 1 slope the factor of safety in the typical winter case (7 foot depth in the creek) is 0.8 which is substantially below the acceptable minimum of 1.5. The factors of safety for the 35 ton and 70 ton shovel loading condition are considerably worse, and also unacceptable.

With the friction angle of 25 degrees which we estimate to represent the upper limit for the loose sand fill, the factor of safety for the existing slope under a normal winter flow, results in a factor of safety of 1.5. This is an acceptable value which we find sufficiently safe. With the creek at the 100 year flood level the factor of safety was computed at 1.32 using the conventional method of slices.

Although a 35 ton shovel is safe, the factor of safety is not high while a 70 ton shovel produces a factor of safety of 0.92, and hence, a failure.

For all practical purposes, the factors of safety for the proposed 2 to 1 cut slope are all unacceptably low and could not be contemplated even for this higher friction angle condition.

E. SLOPE ALTERNATES

1. Present Slope Based on the above analyses it is apparent that the existing slope is only marginally to adequately safe with the estimated strength range assumed. However, even a 35 ton shovel could get into difficulty in local pockets of especially loose sand fill even though the major portion of the fill was sufficiently safe for passage. Moreover, local failure would have to be watched for very closely since tipping of such a large piece of equipment could easily result in progressive bearing capacity failure, and slippage into the creek.

A 70 ton shovel can not be considered for passage along the top of the embankment over the investigated section of sand fill.

2. Proposed Slope The proposed 2 to 1 slope will not be stable for any condition that we have checked including the steady state condition with a relatively low water level. Hence, even with extensive embankment protection such as lining, we can not see much chance for success if the embankment is moved back towards the school and cut on a 2 to 1 slope.

F. EMBANKMENT AND SCHOOL PROTECTION

Information supplied by the District indicates that the 100 year design water level at Station 615 + 00 will be at approximately Elevation 76 feet. This will cause the submergence and saturation of the bottom 8 feet of the loose sand fill. Submergence of granular soils which are under stress has the effect of reducing the bearing capacity by 50 percent if the water level is at the footing bottom, and causing approximately twice the amount of equivalent static settlement to occur.

Proportionately if the lower third of the sand fill is submerged, substantial settlements, perhaps on the order to several inches, could suddenly take place causing significant settlement damage to the overlying school structures. Hence, even though these structures are reportedly temporary in nature it should not be assumed that since settlement has not taken place to date that settlement will not occur at some extended time in the future when a severe flood strikes.

1. Embankment Depending upon the slope inclination finally selected and other influential factors such as the amount of erosion that is tolerable and the location of the top of the slope with reference to the school when construction is finally completed embankment protection may or may not be needed. However, protecting the slope will have a very little effect on the overall stability of the sand fill, since even if a reinforced concrete lining is used, weep holes would permit water to get behind the lining and rapidly saturate and submerge

the fill. Riprap on the slope could act to keep the slope surface in place but due to its light weight when compared with the potential failure mass would not significantly increase the gross stability.

The only way to prevent saturation would be to make a water tight lining with a positive cutoff into the underlying clays in the bottom completely encasing the surface of the top of the slope. However, it is difficult to maintain complete water tightness with such large concrete slabs since minor cracks invariably develop and when placed upon a loose fill embankment are almost certain to develop. The lining would only serve to reduce the rate of inflow into the fill under the school during high flow stages. Also, unless the lining extended upstream and downstream from the school location sufficiently far, water would enter the fill by this means.

In summary, it is our opinion that an economical lining would be relatively ineffective in protecting the school from submergence settlements, would provide little measurable increase in stability to the embankment, but would serve to protect the slope against current erosion.

2. School Shallow foundations are reportedly being used to support the school. Hence, protection to the school against settlement damage during high water levels would require that the foundations be carried down through the loose sand fill which would be subject to settlement. This is a very expensive and arduous undertaking, and of doubtful economic return when protecting temporary structures.

G. RECOMMENDATIONS

1. Embankment Slope Based upon our stability analyses we recommend that the slope be no steeper than 2-3/4 to 1 and be placed no closer to the school than its present location. A preferable slope would be 4 to 1 which would be more stable and tend to minimize some of the slumping of the sand fill below the water level during very high flow states. This would require relocation of the adjacent school. However, since these flood levels occur infrequently it may be less costly to maintain as required and leave the embankment in its present inclination. The relatively flat surface than can be observed approximately two-thirds of the way down the embankment is probably the result of slumping of

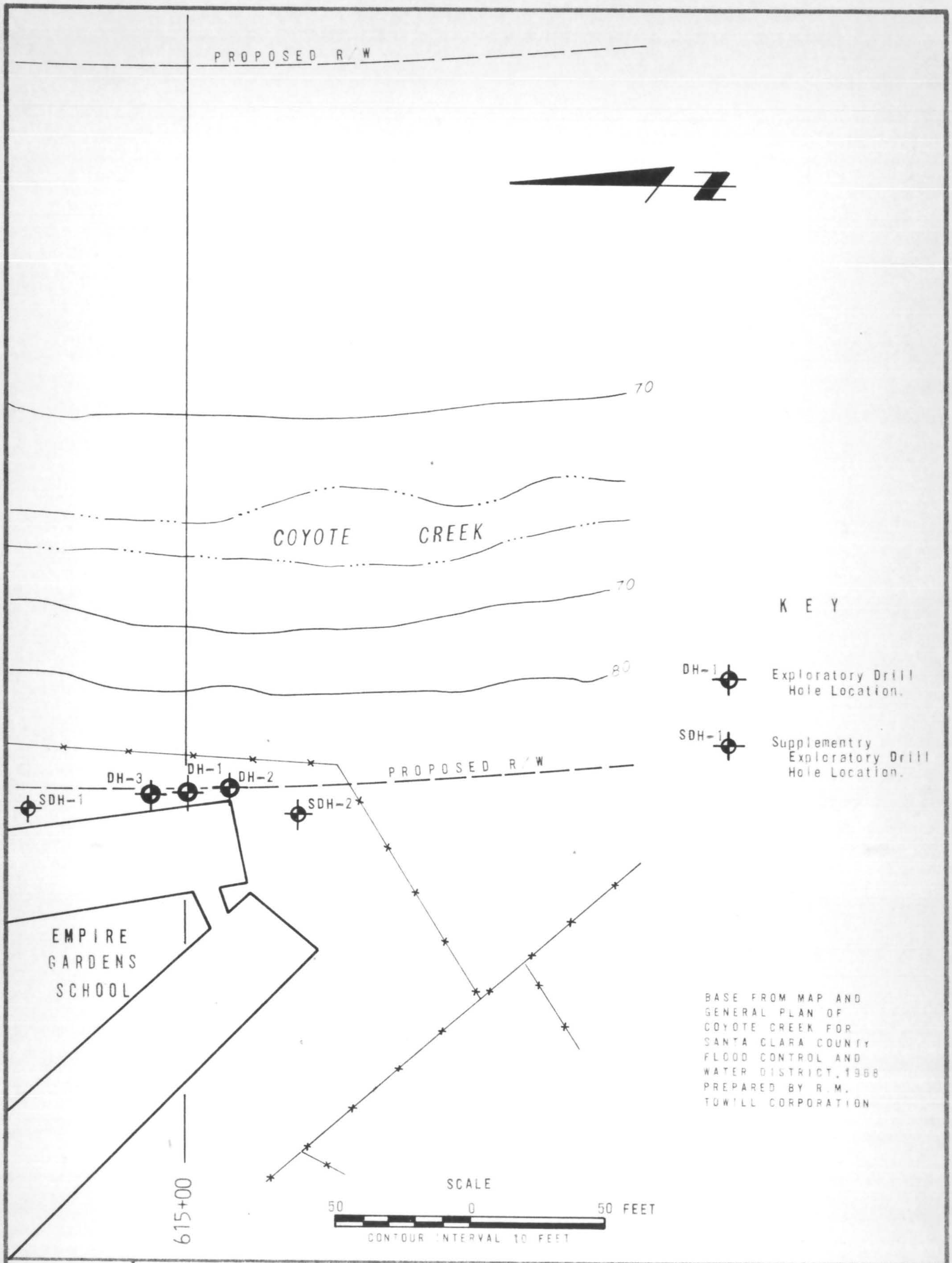
the fill from previous flood levels and bears witness to the expectation that such would continue under more and higher flood levels.

2. Embankment Protection The trees and vegetation presently existing along the embankment serve to knit the slope together in an acceptable fashion and keep the erosion which is potentially severe in loose granular material to a minimum. Hence, if your plans envision removing the vegetation and trees then other suitable slope protection should be considered. However, if the vegetation and trees are left in place and if the cost for maintenance of the slope has not been excessive to date, then in our opinion slope protection will not be needed to minimize erosion of the embankment.

3. School While we would not recommend underpinning the school to protect against submergence settlement, we do recommend that careful consideration be given to any creek construction that is carried out at the site as to its immediate or future effect on the school. In this regard, the loose fill foundation may be looked upon as an adequate foundation medium in its unsaturated condition, but as one having a delayed reaction timing which could produce substantial and costly settlement damages following a very high flood stage in the creek. However, you may find that it is more economical to live with the potential damage resulting from fill submergence and move the temporary structures as required if damaged during a flood. Planting strips should not be used along the Creek since school settlement could result from the irrigation.

H. LIMITATIONS

Between borings, all soil deposits will vary in type, strength, and many other important properties. Our conclusions and recommendations are based partially on the data available to us and our interpretation of that data based on our experience and background. Hence, our conclusions and recommendations are professional opinions and are not meant to indicate a guarantee of a control of nature or a perfection of our art, which is not present.



BASE FROM MAP AND GENERAL PLAN OF COYOTE CREEK FOR SANTA CLARA COUNTY FLOOD CONTROL AND WATER DISTRICT, 1968 PREPARED BY R.M. TOWILL CORPORATION

W.A. WAHLER & ASSOCIATES	SLOPE STABILITY	SITE PLAN		
		PROJECT NO.	DATE	DRAWING NO.
	PALO ALTO • NEWPORT BEACH • CALIF	0276	MARCH 1969	FIGURE 1

APPENDIX A

FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a detailed subsurface exploration program. A total of three 5 inch diameter exploratory holes were drilled on March 12, 1969 to depths ranging from 15 to 50 feet. The holes were advanced by using a truck-mounted continuous flight auger. The locations of the exploratory holes are shown on Figure 1.

The soils encountered were logged continuously by our soils engineer in the field during the exploration and were described in accordance with the Unified Soil Classification System described on the explanatory sheet accompanying the drill hole logs which are included in this Appendix. In addition to the three holes logged, two more holes were drilled to depths of over 20 feet in an attempt to obtain undisturbed samples of the silty clay underlying the sanitary land fill. In these holes the gray silty clay was encountered at 18½ and 19 feet, respectively. A Shelby tube sample could not be obtained since the loose sand and gravel present kept on caving in and blocking the hole.

Disturbed soil samples were obtained from the exploratory drill holes at selected depths. Disturbed soils obtained from sampling or drilling were placed in jars or bags. All samples were transmitted to our Palo Alto laboratory for evaluation and testing.

Standard Penetration Resistance blow counts were obtained by dropping a 140 pound weight having a 30 inch free fall. The sampler was driven 18 inches and the number of blows was noted for each 6 inches of penetration. The blows per foot recorded on the drill hole log represents the second and third 6 inches of penetration totaled together. A standard penetration split spoon (2 inch O.D., 1-3/8 inch I.D.) was used.

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WATER DISTRICT
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UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIEZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.
			GP	POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES.
		GRAVEL WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURE. NON-PLASTIC FINES.
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES. PLASTIC FINES.
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES.
			SP	POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES.
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES. NON-PLASTIC FINES.
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES. PLASTIC FINES.
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN #200 SIEVE SIZE	SILTS & CLAYS	LIQUID LIMIT IS LESS THAN 50	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY.
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS.
			OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY.
	SILTS & CLAYS	LIQUID LIMIT IS GREATER THAN 50	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS.
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS.
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS.
	HIGHLY ORGANIC SOILS		PT	PEAT AND OTHER HIGHLY ORGANIC SOILS.

DEFINITION OF TERMS

GRAIN SIZES

U.S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200 50 16 4 3/4" 3" 6"

SILTS & CLAYS DISTINGUISHED ON BASIS OF PLASTICITY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

MOISTURE CONDITION (INCREASING MOISTURE ⟶)

DRY — SLIGHTLY DAMP — DAMP (PL) — MOIST — VERY MOIST — WET (SATURATED) (LL)

CONSISTENCY

CLAYS & SILTS	BLOWS/FOOT*	STRENGTH‡
Very Soft	0 - 2	0 - 1/4
Soft	2 - 4	1/4 - 1/2
Firm	4 - 8	1/2 - 1
Stiff	8 - 16	1 - 2
Very Stiff	16 - 32	2 - 4
Hard	Over 32	Over 4

RELATIVE DENSITY

SANDS & GRAVELS	BLOWS/FOOT*
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

* Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡ Unconfined compressive strength in tons/sq ft. Read from a pocket penetrometer.

W. A. WAHLER & ASSOCIATES	SLOPE STABILITY	KEY FOR SOIL EXPLORATION LOGS		
	PALO ALTO • NEWPORT BEACH • CALIF.	PROJECT NO.	DATE	DRAWING NO.
		0276	MARCH 1969	---

DRILL RIG CONTINUOUS FLIGHT AUGER	HOLE ELEVATION ---	LOGGED BY AB
GROUNDWATER DEPTH (BELOW GROUND SURFACE) NOT ENCOUNTERED	HOLE DIAMETER 5"	DATE DRILLED 3/12/69

SOIL DESCRIPTION

DESCRIPTION AND REMARKS	COLOR	MOISTURE	CONSIST.	SOIL TYPE	DEPTH (ft)	JARS	SACKS	SAMPLER: SPLIT SPOON	SAMPLER: WATER CONTENT (%)	PENETRATION RESISTANCE (BLOWS/FT.)
SANITARY LANDFILL w/gravel and coarse sand	Dk Brn	Damp to Moist	Loose		0					
					1	x			18	
					2					
					3	x			16	7
Gravelly w/coarse sand		Damp								
					4					
					5					
					6					
					7					
					8					
					9					
Layer of crushed brick, glass and gravel, 18" thick	Med to Dk Brn				10					10
					11	x			10	3
					12					
Loose, sandy	Med Brn				13					
					14					
					15					
					16					

BOTTOM OF HOLE = 15 FEET

NOTE: Drilling abandoned. Hole caved in to 9' after auger was pulled out.

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W.A. WAHLER & ASSOCIATES	SLOPE STABILITY	SOIL EXPLORATION DRILL HOLE LOG		HOLE NO. 1
PALO ALTO • NEWPORT BEACH • CALIF.	PROJECT NO. 0276	DATE MARCH 1969	SHEET NO. 1 OF 1	

DRILL RIG CONTINUOUS FLIGHT AUGER	HOLE ELEVATION ---	LOGGED BY AB
GROUNDWATER DEPTH (BELOW GROUND SURFACE) 28'; ROSE TO 23' in 1 HOUR	HOLE DIAMETER 5"	DATE DRILLED 3/12/69

SOIL DESCRIPTION								SAMPLER		
DESCRIPTION AND REMARKS	COLOR	MOISTURE	CONSIST.	SOIL TYPE	DEPTH (ft)	JARS	SACKS	SPLIT SPOON	WATER CONTENT (%)	PENETRATION RESISTANCE (BLOWS/FT.)
SANITARY LANDFILL; sand and gravel w/zones of trash and debris (old city dump), extends to 21-1/2'	Dk to Med Brn	Damp to Moist	Loose to Med Dense		0					
					1					
					2					
					3	x			14	
					4					
Loose sand and gravel at 5'					5					
					6					
					7					
Loose sand at 7-1/2'	Med Brn	Sl Damp	Loose	SP	8					
					9					
					10					
Gravel and broken stone					11	x			10	
					12					
					13					
					14					
Loose sand					15					
					16					
					17					
					18	x			12	
					19					
					20					
					21					
CLAY w/some silt and fine sand	Med Brn to Gy	Damp to Moist	Firm to Stiff	CL		x			18	
					22					
					23					
					24					
					25					

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W.A. WAHLER & ASSOCIATES	SLOPE STABLIITY	SOIL EXPLORATION DRILL HOLE LOG			HOLE NO. 2
PALO ALTO • NEWPORT BEACH • CALIF.		PROJECT NO. 0276	DATE MARCH 1969	SHEET NO. 1 OF 2	

DRILL RIG CONTINUOUS FLIGHT AUGER	HOLE ELEVATION ---	LOGGED BY AB
GROUNDWATER DEPTH 28'; ROSE TO 23' IN 1 HOUR (BELOW GROUND SURFACE)	HOLE DIAMETER 5"	DATE DRILLED 3/12/69

SOIL DESCRIPTION					DEPTH (ft)	JARS	SACKS	SAMPLER		PENETRATION RESISTANCE (BLOWS/FT.)
DESCRIPTION AND REMARKS	COLOR	MOISTURE	CONSIST.	SOIL TYPE				SPLIT SPOON	WATER CONTENT (%)	
CLAY -- Cont'd.					25					
					26					
Silty w/some fine sand	Gy	Moist	Firm to Stiff	CL	27	x	☒		20	
		▽ Wet			28		☒			
					29					
					30					
					31					
					32					
					33					
					34	x	☒		27	
					35					
					36					
					37					
					38					
					39					
					40					
Silty clay			Stiff		41					
					42					
					43					
					44					
					45	x			34	
					46					
					47					
					48					
					49	x	☒		38	
BOTTOM OF HOLE = 50 FEET					50					

W.A. WAHLER & ASSOCIATES	SLOPE STABILITY	SOIL EXPLORATION DRILL HOLE LOG			HOLE NO. 2
PALO ALTO • NEWPORT BEACH • CALIF.		PROJECT NO. 0276	DATE MARCH 1969	SHEET NO. 2 OF 2	

DRILL RIG CONTINUOUS FLIGHT AUGER	HOLE ELEVATION ---	LOGGED BY AB
GROUNDWATER DEPTH (BELOW GROUND SURFACE) NOT ENCOUNTERED	HOLE DIAMETER 6"	DATE DRILLED 3/12/69

SOIL DESCRIPTION					DEPTH (ft)	JARS	SACKS	SAMPLER		PENETRATION RESISTANCE (BLOWS/FT.)
DESCRIPTION AND REMARKS	COLOR	MOISTURE	CONSIST.	SOIL TYPE				SPLIT SPOON	WATER CONTENT (%)	
SANITARY LANDFILL; sand and gravel w/zones of trash and debris (old city dump), extends to 20-1/2'	Dk to Med Brn	Damp	Loose		0					
					1					
					2					
					3					
					4					
					5	x			14	
					6					
					7					
					8					
					9					
Large pieces, up to 10", of broken rock or concrete					10					
					11	x			11	
					12					
					13					
					14					
					15					
					16	x			17	
					17					
					18					
					19					
					20					
CLAY; silty	Med Brn to Gy	Damp to Moist	Stiff	CL	21	x			19	
					22					
BOTTOM OF HOLE = 23 FEET					23					
					24					

W.A. WAHLER & ASSOCIATES	SLOPE STABILITY	SOIL EXPLORATION DRILL HOLE LOG		HOLE NO. 3
		PROJECT NO. 0276	DATE MARCH 1969	SHEET NO. 1 OF 1

PALO ALTO • NEWPORT BEACH • CALIF.

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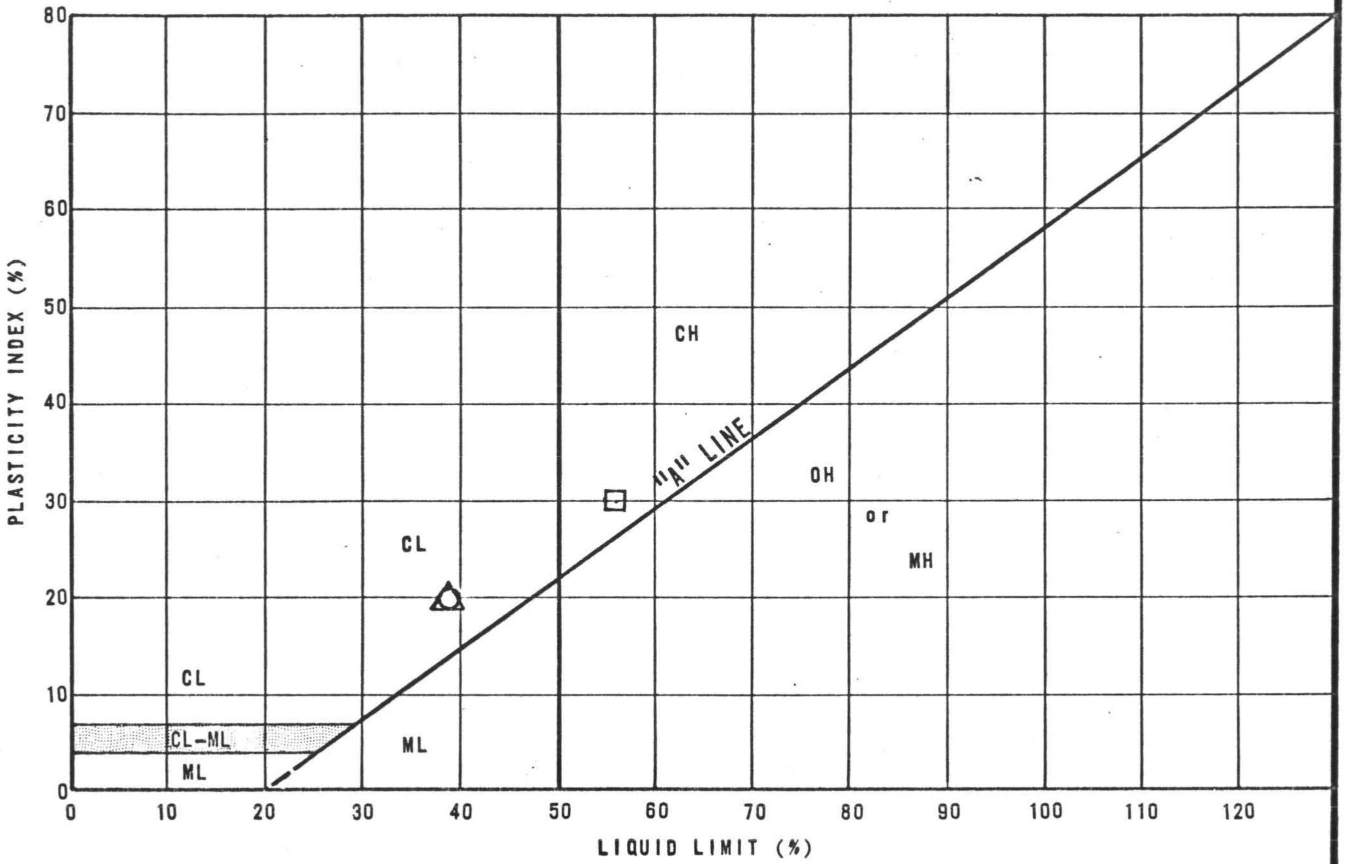
APPENDIX B

LABORATORY INVESTIGATION

The laboratory investigation was programmed to determine the pertinent index and engineering properties of the foundation soils underlying the site. Testing results combined with observations made during the subsurface exploration were used as the main criteria for determining the engineering suitability of the surface and subsurface materials encountered.

Natural water contents were determined for all of the collected samples to determine the variation of moisture within the explored subsurface profile. The results are presented on the drill hole logs. In addition, Atterberg Limits tests were performed on the silty clay obtained from a depths of 28, 34 and 49 feet to determine the plasticity characteristics of the soil. The results are given in Figure 2.

PLASTICITY CHART



PLASTICITY DATA

KEY SYMBOL	HOLE NUMBER	DEPTH (feet)	NATURAL WATER CONTENT W (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	LIQUIDITY INDEX $\left(\frac{W - PL}{LL - PL}\right)$	UNIFIED SOIL CLASSIFICATION SYMBOL
○	2	34-35	27.2	19	39	20	0.4	CL
△	2	27-28.5	--	19	39	20	--	CL
□	2	49-50	38.4	26	56	30	0.4	CH

740-3

SOIL MECHANICS and FOUNDATION ENGINEERS Inc.

PALO ALTO • NEWPORT BEACH • CALIF.

CREEK STABILITY, COYOTE
CREEK & EMPIRE GARDENS SCHOOL
SAN JOSE

ATTERBERG LIMITS - PLASTICITY DATA

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0276	March 1969	Figure 2

APPENDIX C

STABILITY ANALYSES

The stability of a slope is dependent on forces acting on potential sliding surfaces and on the shear strength of the soil. Forces that the slope must resist include the weight of the soil, unbalanced water pressure, seeping water, and the weight of spoil banks, equipment and materials. The stability of a slope may also be affected by future construction on or above the slope and by changes in internal pore pressures and outside water levels. Field experience with past slides indicates that for purposes of analyses the failure surface can be assumed to be circular in section and infinite in width.

Three methods of analyses were involved using the IBM 7094 computer programs developed by this firm: 1) conventional method of slices (Brown Method), 2) Bishop Method, and 3) Janbu Method. Results range from the simplifying, expeditious assumptions concerning the side forces between slices made in the first method to the more accurate and rigorous approaches made in the Bishop and Janbu procedures.

The factor of safety is defined as the ratio of the shearing strength available to the shearing strength necessary to prevent instability. In other words, the slope would be thrown into a condition of incipient failure if the shearing strength were suddenly divided by the factor of safety.

1. CONVENTIONAL METHOD OF SLICES (BROWN METHOD)

After a circular failure arc is assumed, the soil mass above the sliding surface is divided into a series of vertical slices. Forces acting on each slice are earth pressures on the sides of the slice, water pressures on the sides and bottom, effective earth forces across the sliding surface, and cohesion along the sliding surface. For expediency, it is assumed in this method that the forces acting on the sides of the slices are in equilibrium (equal and opposite in direction to each other) and only the forces on the base of the slices need be considered. The weight vector consists of the saturated weight of soil above the water table plus the submerged weight of soil below the water table. The normal component of the weight vector is reduced by the pore pressure acting on the base area of the slice.

The resisting force consists of the sum of the cohesion along the bottom of the slice and the normal component of the weight times the tangent of the angle of shearing resistance. The driving force consists of the tangential component of the weight vector. The factor of safety is determined by comparing the moment of resistance offered by the cohesion and friction on the failure surface to the moment of the soil mass about the center of the assumed failure arc. Various trial failure arcs are analysed until a minimum factor of safety is obtained.

2. BISHOP METHOD

This method, a more refined variant of the conventional method of slices, considers all of the above. In addition, the equations of equilibrium are satisfied using successive approximations. Trial failure circles are analyzed holding the slices in complete static equilibrium. Factors of safety are again obtained by comparing the driving and resisting moments and by this method are generally 10 to 15 percent higher than that computed by the conventional method. The difference is particularly pronounced for slopes subject to high pore pressures and deep failure circles.

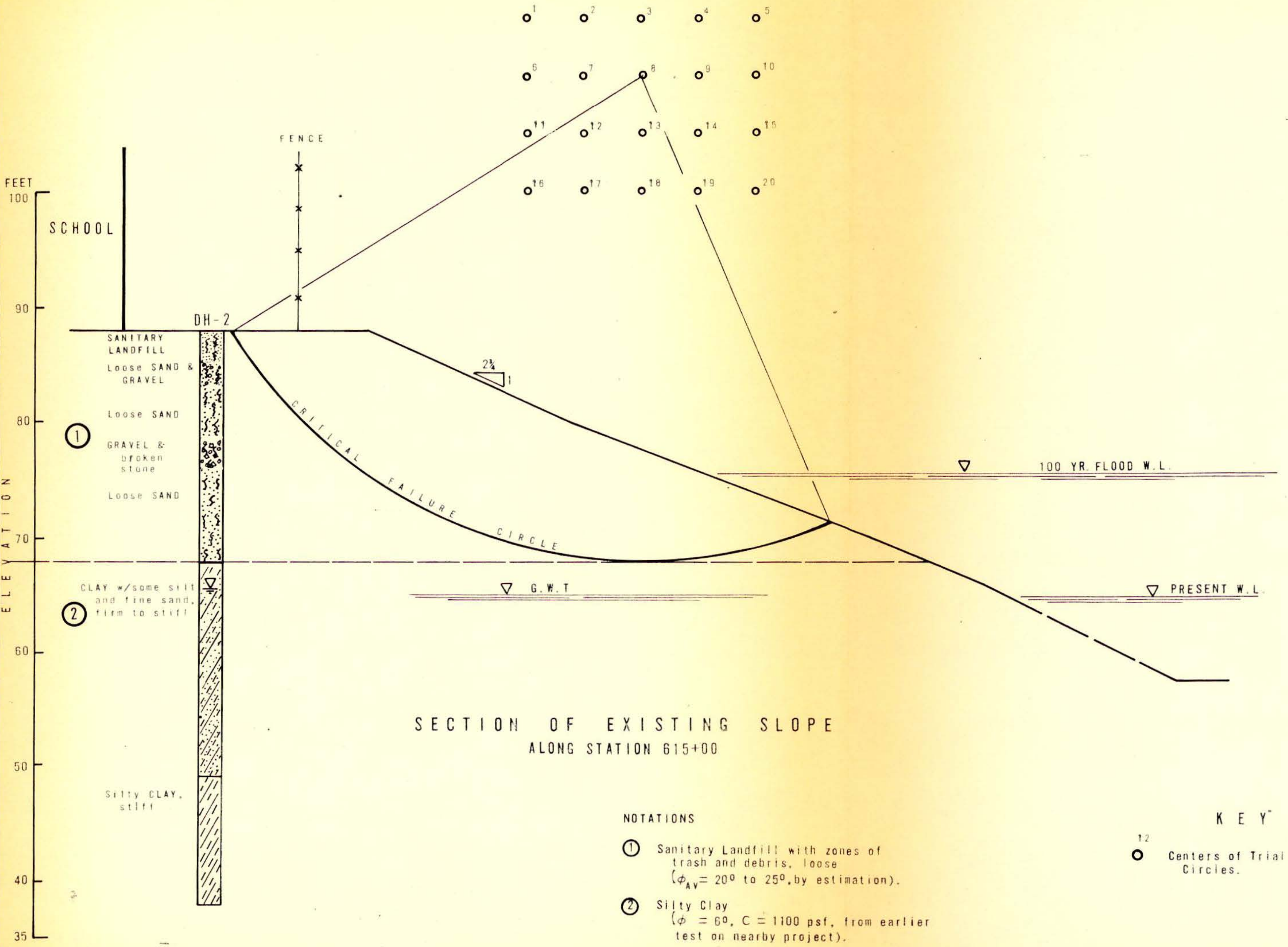
3. JANBU METHOD

This method also considers equilibrium for all internal forces between slices but permits any configuration of failure surface to be considered, not just cylindrical. Janbu's method has been further developed by this firm to permit inclusion of the water forces on the sides of the slice and to permit horizontal body forces. Thus, special applications such as an approximation for seismic loadings and the use of nonvertical slices can be handled.

4. APPLICATION OF SLOPE STABILITY ANALYSIS

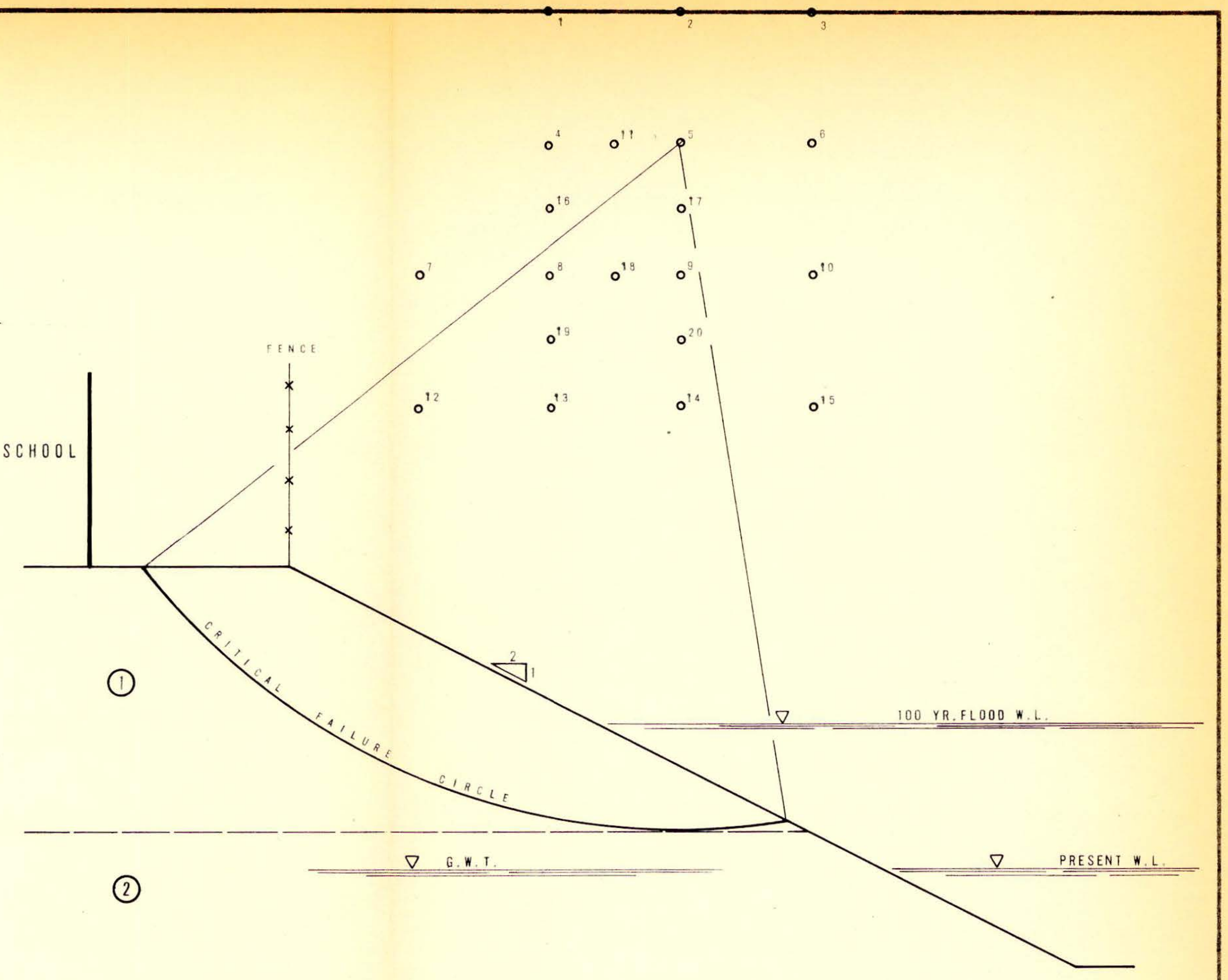
Our stability investigation considered three distinct cases for each of the two different slope configurations. The existing slope (2-3/4:1) and the proposed 2 to 1 cut stationing line 615 + 00 were both analysed for the steady state condition; and for the presence of 35 and 70 ton shovels on the top of the slope. Two sets of soil properties were used for each slope; 1) friction angle of fill = 20°, 2) friction angle of fill = 25°.

The results of the stability analyses for the various conditions are shown in Table I in the text of the report and the sections analysed in Figure 3.



SECTION OF EXISTING SLOPE

ALONG STATION 615+00



SECTION OF PROPOSED 2 TO 1 CUT

ALONG STATION 615+00

NOT TO SCALE

NOTATIONS

① Sanitary Landfill with zones of trash and debris, loose ($\phi_{AV} = 20^\circ$ to 25°, by estimation).

② Silty Clay ($\phi = 6^\circ$, $C = 1100$ psf, from earlier test on nearby project).

KEY

$\overset{12}{\circ}$ Centers of Trial Failure Circles.

W. A. WAHLER
& ASSOCIATES

SLOPE STABILITY
STATIC, STEADY STATE CONDITION
PALO ALTO • NEWPORT BEACH • CALIF.

SLOPE STABILITY STUDY

PROJECT NO.	DATE	DRAWING NO.
0276	MARCH 1969	FIGURE 3